# REMEDIATION OF CONTAMINATED MARINE SEDIMENTS USING MUDCRETE – A CASE HISTORY

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## ABSTRACT

Remediation of the second-highest priority contaminated site in New Zealand has provided an opportunity to remove and stabilise contaminated marine sediments from the Calwell Slipway Basin, Port Nelson. Other drivers for the project included restoring navigability and extending an existing reclamation. The new reclamation was constructed with dredged marine sediments mixed with Portland cement and powder activated carbon to create mudcrete. The addition of cement and powder activated carbon increases the strength of the sediments and minimises leaching of contaminants. Environmental and engineering design components are discussed including results of the laboratory bench trial undertaken to determine appropriate dosage rates, full-scale production trials, and results achieved during construction. Geotechnical design was governed by seismic loading and the need to mitigate liquefaction of the existing reclamation fill and marine sediments beneath the mudcrete.

## 1. INTRODUCTION

The Calwell Slipway is located on reclaimed land, with reclamation around the slipway commencing circa 1957, and continuing to around 1969. Calwell Slipway is New Zealand's third largest slipway, providing a location to vessels of all kinds to perform hull repairs and maintenance. Port Nelson requires the seabed levels within the harbour and slipway be maintained to ensure navigability of the port and to support operation of the slipway. Contamination of the Calwell Slipway is thought to have begun in 1970 due to inadequate containment and disposal of wash water during vessel servicing. The antifouling process and maintenance operations resulted in contamination of the marine sediments, with tributyltin (TBT) and copper concentrations, amongst other contaminants, exceeding the allowable contaminate limits within the disposal permit for Tasman Bay. Alternative treatment and disposal of the dredged sediments was required, comprising treatment of the contaminated sediments with cement and powder activated carbon, creating mudcrete. The treated sediments were then disposed within a dedicated reclamation zone within the Calwell Slipway basin, extending the reclaimed land along the slipway inlet, creating approximately 5000 m$^2$ of additional land storage and parking area for Port Nelson, refer Figure 1 for approximate layout.


**Figure 1: Port Nelson - Calwell Slipway**

# 2. GEOLOGY

Regional geological maps show the site to be underlain by land reclaimed from Nelson Haven. The reclamation was created by pumping sand and silt from the floor of the Haven into settling ponds bounded by embankments made of locally quarried Port Hills Gravel. The dredged material was placed directly on top of Holocene aged alluvial and beach deposits, which forms the geology beneath Nelson township. The Holocene deposits are further underlain by Pliocene aged Port Hills Gravel of the Tadmor Group, comprising poorly to moderately well sorted clay bound gravel.

Nelson is located at the base of the Bryant Range, which forms part of the eastern limb of the 30 km wide Moutere Depression. The depression is fault bounded to the east by the south-west to north-east trending Waimea-Flaxmore fault system. The site is located 2.5 km west of the Flaxmore fault. The Waimea-Flaxmore fault system extends north-east from the Alpine fault, through the eastern side of Tasman District and into Nelson City and Tasman Bay. Movement recurrence of the Waimea and Flaxmore faults are estimated (by GNS) to be around 6000 years, approximately equal to the last time rupture occurred along the fault (Johnson et al, 2013). The Alpine Fault is located approximately 40 km south-east from Nelson and has ruptured four times in the past 900 years at various locations along the 600 km long Fault, with the potential for intense seismic shaking in Nelson depending on the rupture location.

# 3. GEOTECHNICAL DESIGN

## 3.1 GEOTECHNICAL INVESTIGATIONS

Site investigations were undertaken to determine the geotechnical characteristics of the site and develop a comprehensive ground model. Investigations through the existing reclaimed land were primarily focused on gathering data for susceptibility to liquefaction during a seismic event, with investigations located on the Calwell Slipway jetty to determine the thickness of the in-situ marine sediments overlying the Port Hills Gravel. Site investigations comprised four sonic machine drilled boreholes and six CPTu soundings accompanied by two downhole seismic shear wave velocity tests. Samples from the boreholes were sent to the laboratory for geotechnical testing.

The investigations confirmed the type of material used to construct the rockfill bund that contained the hydraulic fill and provide permanent stability to the edge of the reclamation. The thickness of the reclamation fill encountered was between 2.8 m to 4.6 m, underlain by up to 6 m of normally consolidated estuarine deposits comprising loose to very loose sandy silt, silty sand and gravelly sand. Boreholes advanced from the Calwell Slipway jetty encountered a 2-3 m thick layer of recent marine muds overlying the marine sediments. This marine mud is thicker under the jetty due to maintenance dredging operations being unable to reach and remove the material.

## 3.2 MUDCRETE DESIGN

Bulk samples of the in-situ marine sediments surrounding the slipway were collected for stabilisation trials. Bulk samples were blended with cement and powder activated carbon at a testing laboratory, herein referred to as bench trials. The blended samples included dry cement blends and a wet grout mixture. Two phases of bench trials were undertaken; the first phase was done primarily for contamination purposes and to determine the additive content required to contain the contaminants, the second phase was to confirm the strength and validity for geotechnical design. Samples blended during the second phase of trials were transferred into a seawater bath until they were tested to replicate site conditions.

Some strength testing was undertaken in the first phase of the laboratory bench trials, with UCS testing done on selected blends. The results indicated a cement dosage of 60 kg/m3 (approximately 4% of sampled bulk density) will increase the undrained shear strength ($S_u$) from 20 kPa to 110 kPa after 7 days. Higher dosage rates resulted in higher strength, with 100 kg/m3 (6%) increasing $S_u$ to 220 kPa.

Following completion of the first phase of testing and acceptance of the cement stabilisation method, further geotechnical laboratory testing was required for detailed design. This comprised three consolidated undrained triaxial (CU) tests on treated samples to determine the effective stress parameters for the long-term behaviour of the revetment slopes. Unconsolidated undrained triaxial (UU) tests on treated samples were tested at 7 and 28 days to determine undrained shear strength for short term design, and particle size distribution and bulk density testing on natural material to provide baseline values for construction.

UU test results showed the dry cement additive produced higher strength than the grout mixture. The reasons for this are not completely known and could be due to several factors, such as cement reducing the moisture content of the sample through hydration, water to cement ratio during blending, in-situ moisture content of the sample, or blending errors. The results from the dry cement samples indicated that a dosage of 80 kg/m3 (4% to 5% depending on sample bulk density) would provide $S_u$ values of 115 kPa after 7 days, increasing to 150 kPa after 28 days, with average strength gain of 39%.

On this basis, the average design undrained shear strength chosen was 150 kPa, with a minimum of 100 kPa allowing for some variability in strength. Results of the UU stabilisation bench trials are shown in Figure 2.

CU test results were completed on dry cement blends only, with cement a dosage of 80 kg/m$^3$. Based on the results, the design effective stress parameters were selected as c'=10 kPa, phi'=34°.

The bulk density of the materials sampled during the second phase was much higher on average than the first phase. On inspection of the results, the change in bulk density was attributed to sample location. Two of the three sample sites were located alongside the existing revetment slopes and would have been subject to maintenance dredging, with the remaining site located near the existing jetty adjacent to the Calwell Slipway; this site also had a much higher fines content of 48%.

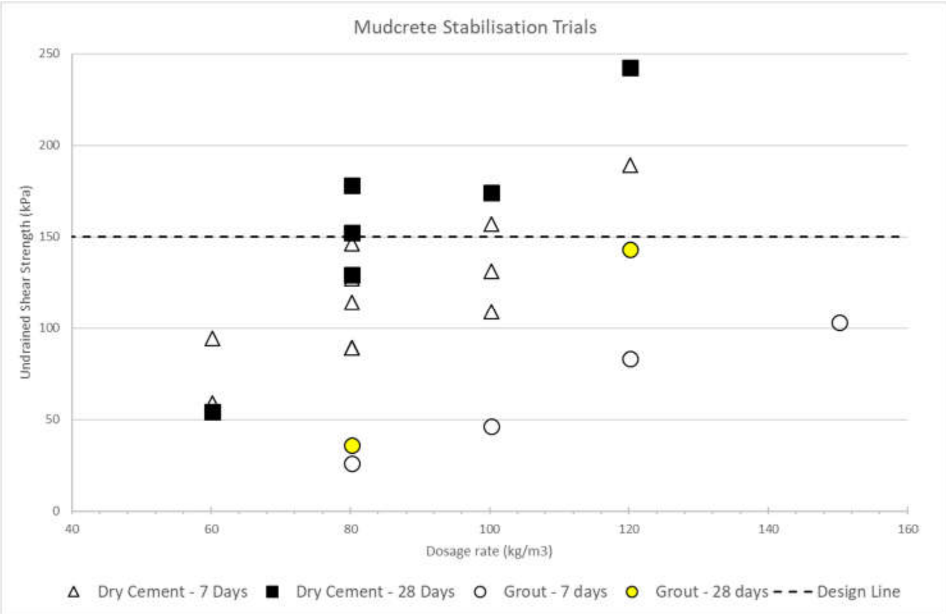


**Figure 2: UU Test Results (Trials)**

### 3.3 LIQUEFACTION ASSESSMENT

Site investigations confirmed the presence of soils susceptible to liquefaction, i.e. loose, saturated, recently deposited sandy soils. The known soil stratigraphy, accompanied by the seismic hazard in Nelson, puts the site at risk of liquefaction-induced damage in a seismic event, including lateral spreading along the existing reclamation revetment slopes. Results of the analyses indicated the site was highly susceptible to liquefaction during an earthquake, classified as 'High risk' under the liquefaction potential index (LPI). Estimated free field settlement was in the order of 50 to 100 mm, assuming the rockfill containment bunds surrounding the reclamation remained stable and were sufficiently strong enough to prevent lateral spreading.

The integrity of the rockfill bunds is not well known, investigations to assess liquefaction potential across the Port Nelson area in 2013 indicated bund material to be loose to medium dense gravel consistent with the material being 'end dumped' into the harbour. Investigations undertaken in 2015 alongside the Calwell basin encountered loose gravelly sandy silt, this material is more likely a general bulk fill than containment bund material. The liquefaction triggering analyses indicate this layer is partially susceptible to liquefaction under ULS seismic loading. Following the investigations and our assessment of the liquefaction potential, the containment bund has been assumed liquefiable.

### 3.4 SLOPE STABILITY

Stability analyses were undertaken for the pre-existing site conditions and the proposed reclamation. Bathymetry and topographical survey data indicated the pre-existing slopes were in the order of 7.5 m high. Removal of the contaminated marine sediments and underlying liquefiable estuarine deposits increased the slope height to approximately 11 m.

Stability assessment of the revetment slopes was undertaken for both static and seismic conditions. The purpose of the static stability analyses was to determine an appropriate slope angle for long-term performance, with a factor of safety of 1.5 required under a 10 kPa surcharge loading. To achieve an adequate factor of safety, the final design slope around the outside of the reclamation area is 1V:1.5H (34 degrees). Erosion protection was added with graded rock armour placed in layers around the reclamation.

Seismic slope stability was analysed under ULS seismic conditions to determine the performance and behaviour of the mudcrete block. The mudcrete block was modelled using total stress parameters during seismic loading, using the minimum design $S_u$ of 100 kPa. Through analysis it was determined that leaving the estuarine deposits in place beneath the proposed reclamation would result in flow type failures, i.e. a factor of safety of less than 1.0 achieved post seismic with liquefied soil conditions, which would result in excessive lateral and vertical deformation, with the potential to impact the Calwell slipway. To reduce the seismic risk, the liquefiable sediments were to be dredged out and mixed in with the contaminated soils, stabilised, then placed within the reclamation area, creating a block of mudcrete down to the Port Hills Gravel. Further analysis was undertaken whereby the in-situ estuarine deposits and reclamation fill against the mudcrete block was assumed to liquefy during a seismic event, in order to determine the level of deformation that could be expected. Deformation analysis was undertaken using a sliding block method to determine the yield acceleration, i.e. acceleration required to reach a factor of safety equal to 1.0, with deformation estimated using methods by Ambraseys and Menu (1988), Jibson (2007), Bray and Travasarou (2007). With the mudcrete block extending down to the Port Hills gravel unit, the deformation across the reclamation zone was estimated to be in the order of 50 mm. The design profile is illustrated in Figure 3.

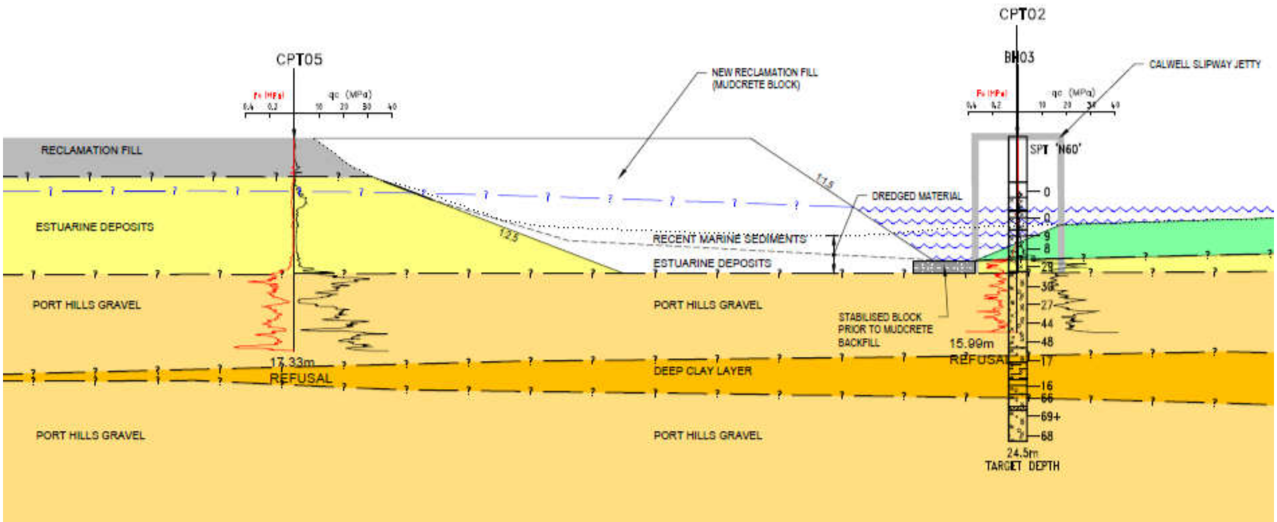


**Figure 3: Mudcrete reclamation**

# 4. CONSTRUCTION

## 4.1 CONSTRUCTION METHODOLOGY

All dredging works were undertaken using an excavator fixed to a pontoon supported on spud legs. The excavator was fitted with Seatools electronic dredging software (DipMate), allowing the excavator operator to accurately determine the position of the excavator bucket, boom and dipper, with all data transmitted in real-time.

Dredged material was loaded directly into hopper barges. Once the hopper barges were full, a tugboat manoeuvred the barge to the unloading facility where the sediments where transferred directly into the pugmill for stabilisation with Portland cement and powder activated carbon (PAC). Cement and PAC was added via variable speed screw feed, with dosage automatically adjusted based on the measured mass passing over the primary belt conveyor. Once processed, the treated material was transferred immediately into a truck and moved to the reclamation zone. The material was generally end tipped on land and then uplifted with a long-reach excavator and placed as directly as possible onto the seabed to prevent segregation of the mudcrete as it passed through the water column. Once the mudcrete level was above sea level, the material was generally placed on the slopes to allow further advancement of the reclamation.

## 4.2 SITE TRIAL

A site trial was completed within the reclamation zone. The purpose of the trial bund was to confirm the dosage rate, blending consistency, and placement methodology for the mudcrete. The trial bund was constructed using a dosage rate of 5% cement (approximately 80kg/m$^3$) and 0.3% power activated carbon, based on measured bulk density. This was measured directly within the processing plant, with the dosage rates adjusted within the control room when necessary.

Investigation boreholes and CPTu's were undertaken to assess the overall performance and condition of the placed mudcrete once the material had been placed for 28 days. The results of the investigations show weakened material below mean sea level (MSL), approximately 5.0m below finished platform level of RL 5.3 m, with lenses of weaker and poorly cemented material in isolated zones. Push tube samples were collected in this weaker material, with CU triaxial testing undertaken. The results confirmed that the effective stress design parameters were being achieved, as shown in Figure 5. The site investigations also encountered marine sediments at the base of the mudcrete, within the zone that dredging was taken down to the top of the Port Hills gravel unit, i.e. some material had not been completely treated.

The site trial provided valuable information to the performance of the material and the methodology. Revisions to the design and construction methodology were made prior to the full-scale production phase. The main changes are as listed below:

- Increase cement dosage to 6% of bulk density (approximately 100 kg/m$^3$) to provide more consistent strength below water;
- Placement procedure was updated to minimise segregation of the mudcrete as it went through the water column. The method of letting the mudcrete flow down the made slopes and out of the bucket was replaced by leaving the mudcrete within the bucket to the placement level, then dumping the mudcrete at the desired location;
- Total dredge depth was achieved as indicated by the calibrated equipment within the excavator, however in-situ testing indicated that a layer of weak material existed. The dredging depth was to be confirmed by operator immediately prior to placing mudcrete to minimise any weak / soft material flowing into the dredged excavation.
- During the trial, water was being added through the conveyor system to make the material more workable and flow from the excavator bucket more readily. The material dredged during early phase of the trial bund construction comprised the contaminated marine muds, i.e. finer grained material with high water contents. UU testing within this material indicated a moisture content after 7 days of 70-80%, with little reduction in moisture content at 28 days. The additional water in the finer grained material likely had a negative effect with respect to the water cement ratio and the resultant strength gain.

### 4.3 QUALITY CONTROL DURING CONSTRUCTION

Following the trials, full-scale production and reclamation was commenced. Quality control testing was required for the duration of the project with treated marine sediments collected directly from the pugmill conveyor at specified production intervals. The material extracted from the processing plant showed a consistent mixture, with the laboratory test results consistently meeting the design requirements. Lower strengths were recorded in material with moisture content above 70%, which was more typical in the finer grained contaminated sediments near the surface. Figure 4 presents all the UU test results undertaken during trials and production of the mudcrete during the reclamation project.

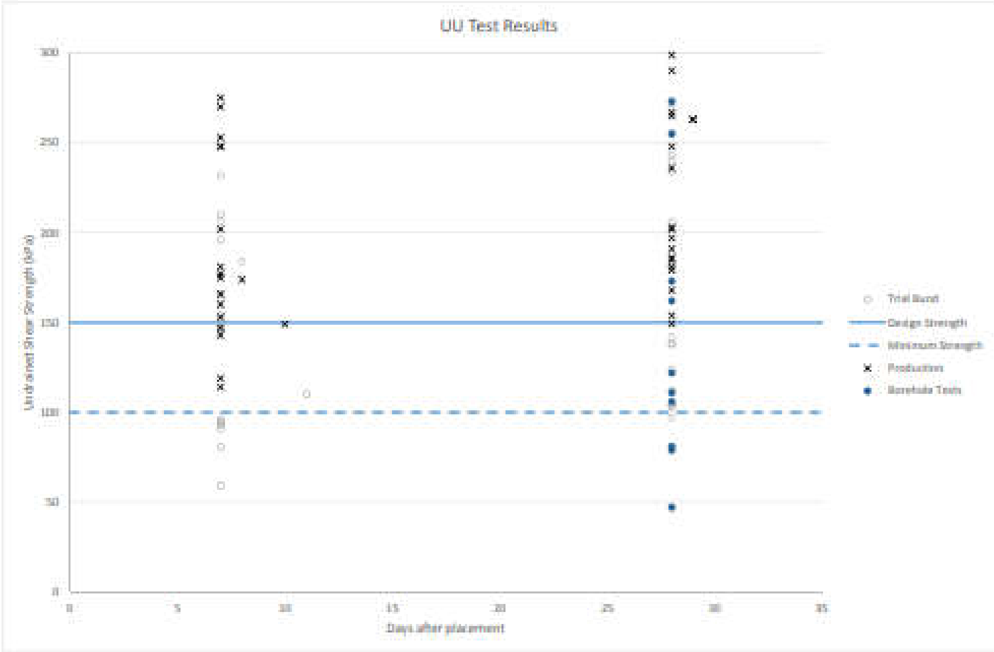


**Figure 4: UU Test results during construction**

CU triaxial testing was also undertaken at specified intervals from samples collected off the pugmill conveyer. All samples tested met the design requirements, including the samples collected during the site trial phase. Figure 5 presents the p'-q plots of all the CU triaxial testing completed during the project.

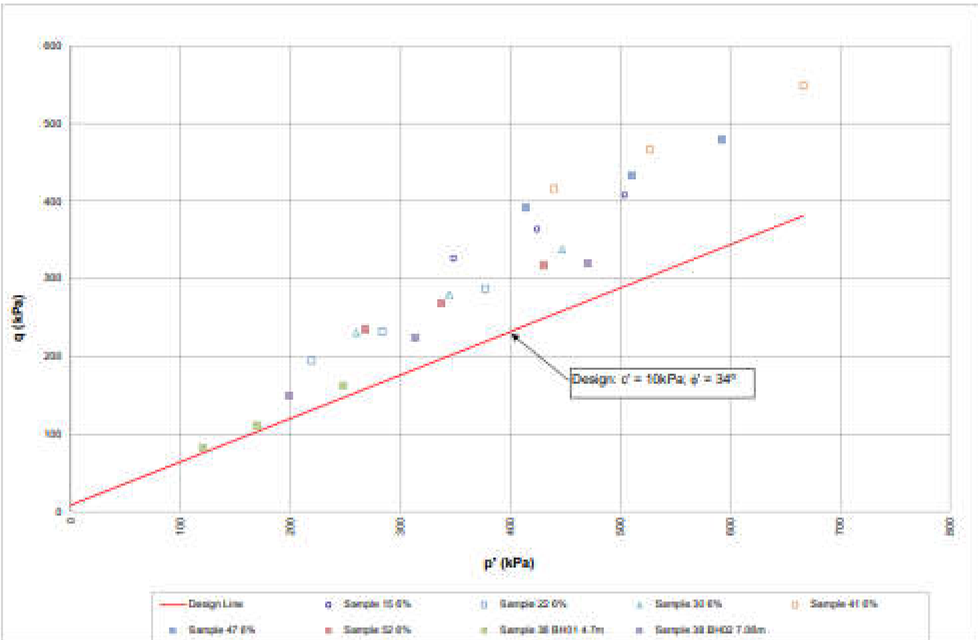


**Figure 5: CU Test results**

### 4.3.1 Insitu testing results

Additional CPTs were undertaken based on results from the trial bund. The CPTs were undertaken to gain a strength profile below MSL and the effects of increasing the cement dosage to 6% of the bulk density. The results indicated that strength gain was quicker with the increased dosage, however comparison between the trial bund area with 5% cement and the zones with higher 6% dosage indicated the final undrained shear strengths were similar at completion of the project. Results from the UU triaxial tests were used to calibrate the cone factor ($N_{kt}$) value used to estimate undrained shear strength from the CPTu's. An average value of 14 was calculated over seven individual samples, with $N_{kt}$ values varying from 10 to 16. For illustrative purposes, an $N_{kt}$ value of 14 has been adopted. Figure 6 presents the results of two CPTu's located near each other, tested approximately three months apart. The results show an increase in mudcrete strength with 5% cement and 6% cement dosage rates over the three month period.

Weaker zones with lenses of material below the design strength required (150 kPa) are also shown in Figure 6. The weaker zones were non-continuous and could be a result of several influencing factors, such as water ingress, processing and placement. UU testing was also undertaken on selected samples from the boreholes, refer Figure 4, confirming the poorly cemented core did not achieve the specified design strength of 150 kPa, however the strength was significantly greater than non-stabilised sediments.

Insitu testing results and core logs also show inconsistency in achieving the design depths. The results indicate that a basal layer of estuarine deposits was not completely treated, refer Figure 6. From the insitu test results, the layer is typically no more than 0.5 m thick and ranges from negligible amounts up to approximately 2.0 m thick across the reclamation site. The thickest zones were around the trial bund area, where a wide trench was excavated down to the design dredge level, potentially allowing disturbed material to flow into the excavation.

Additional stability modelling was undertaken to assess the overall impact this layer has on the reclamation during a ULS seismic event. The results indicated an increase in displacement from 50 mm to 250 mm. The modelling assumed a theoretical worst case of a 2.0 m thick continuous layer of un-cemented material. The effects of the mudcrete bund and the change in effective stress were not taken into account for estimating the liquefied shear strength ratio ($\tau/\sigma_v$`) of the layer.

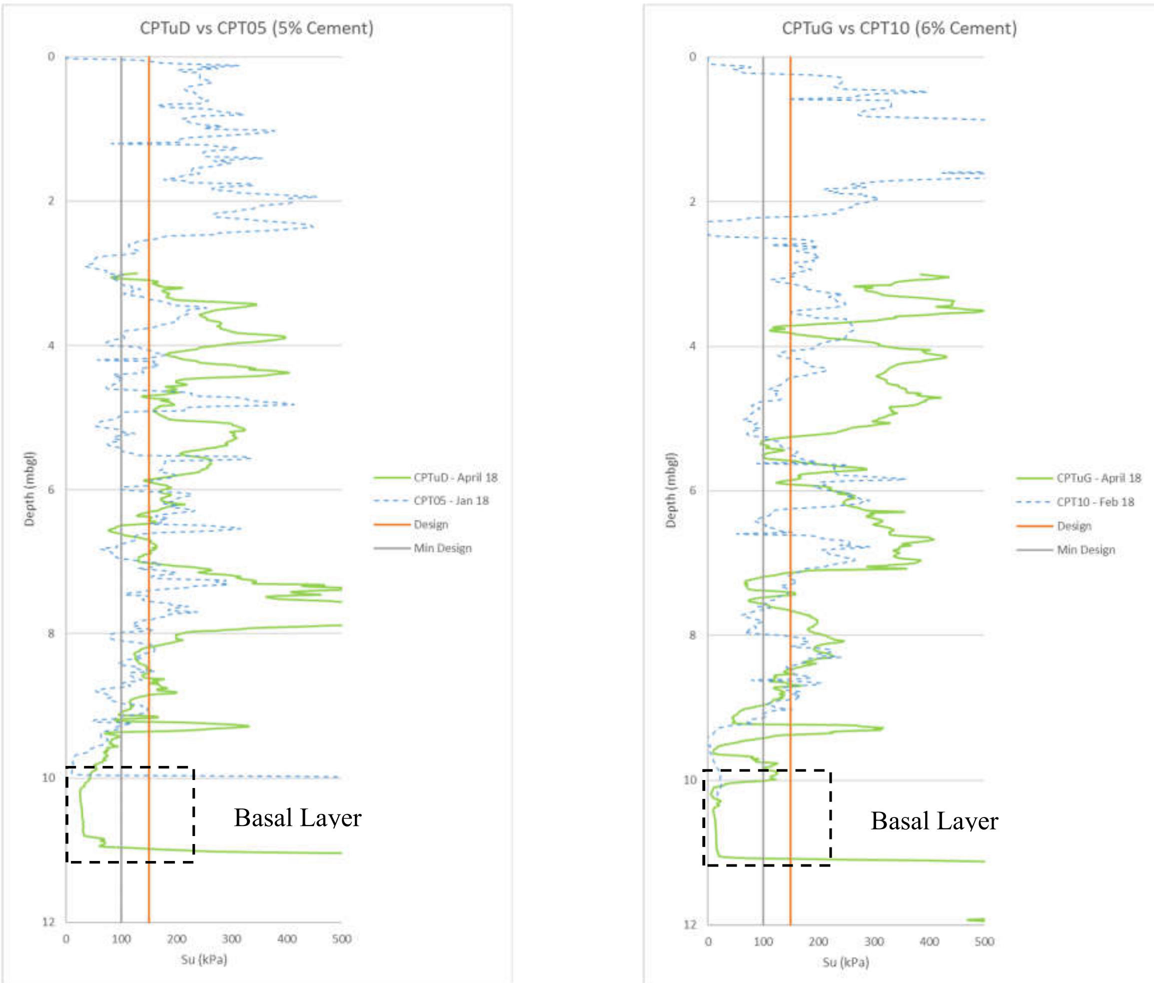


**Figure 6: Comparison between dosage rate and testing 3 months apart**

# 5. CONCLUSION

The mudcrete reclamation has provided Port Nelson with an additional 5000 $m^2$ of useable land for storage and port operations, whilst achieving the overall objective of cleaning up contaminated sediments from the Calwell Slipway basin. The final dredge volume was 32,516$m^3$, with 58,781 tonnes of material processed and placed into the reclamation zone. The project also restored navigability to the Calwell Slipway basin.

The marine sediments processed through the pugmill produced a well-blended material. Quality control testing was undertaken at regular intervals throughout the project duration with samples sealed within PVC tubes for UU triaxial testing to confirm the 7 day and 28 day undrained shear strengths. Results consistently achieved the specified design strengths and confirmed the results achieved during laboratory bench trials. The processed mudcrete exceeded the specified design requirements above sea level, whilst generally achieving the minimum design undrained shear strength of 100 kPa below sea level. The reduced strength below sea level was determined through CPTu testing and grab samples during borehole investigations, and is shown below approximately 4.5 m (0.5 mRL) in Figure 6. Localised weaker zones were encountered across the reclamation but were generally limited to isolated zones which were remediated during construction. The reduced strength below sea level could be the result of a combination of effects, such as water ingress, processing, placement methodology, cement segregation, etc.

Final dredge depths highlighted irregularities between the dredged surface taken from the excavator and the insitu testing, with up to 2 m of material encountered beneath the mudcrete considered as not completely treated. The final dredge depths were checked against the computer software within the excavator to confirm the validity of the design dredge depth. This was achieved by visual confirmation of the Port Hills gravel unit. It is not known whether the dredge depth was checked prior to placing the mudcrete into the excavation, which could account for some localised disturbed material from the sides of the excavations being not completely treated. The not completely treated basal layer was encountered across the reclamation area, typically no more than 0.5 m thick, and up to 2.0 m thick in the worst affected areas. The result of this basal layer increased the overall risk of lateral deformation during an earthquake, with the displacement estimated to be to be in the order of 250 mm across the reclamation.

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